Structural Problems

Part II

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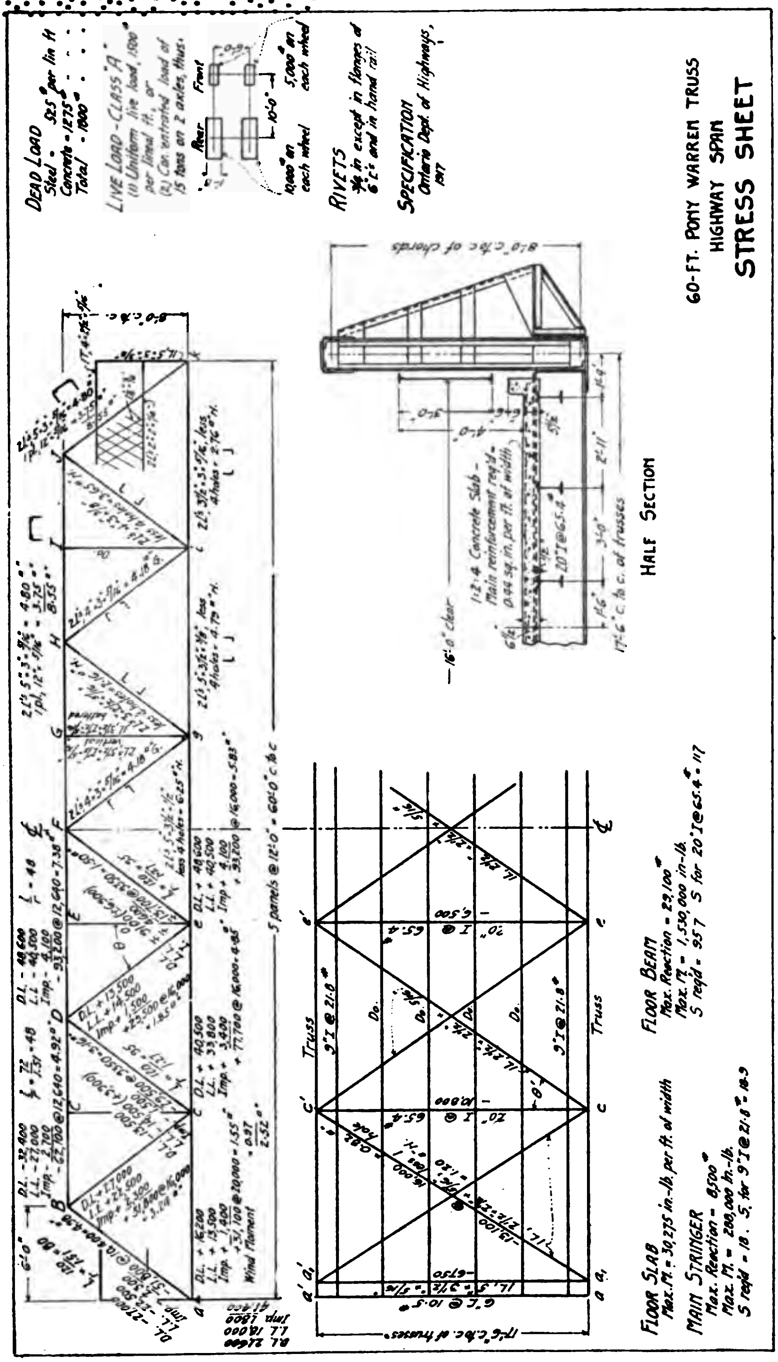


FIG. 1—STRESS SHEET FOR 60-FT. PONY WARREN TRUSS HIGHWAY SPAN

DESIGN OF A PONY WARREN TRUSS HIGHWAY SPAN

Data

Span. 60 ft. centre to centre of bearings; 5 panels of 12 ft. each.

Depth. 8 ft. c. to c. of chords.

Width. 17 ft. 6 in. c. to c. of trusses; 16 ft. clear roadway between handrails at level of hub.

Floor. Reinforced concrete slab. No end floor beam.

Loads. Class "A" live loading, Specifications for Steel Highway Bridges, Department of Public Highways, Ontario, 1917, consisting of (1) a uniform load of 100 lb. per sq. ft., *or* (2) a concentrated load of 15 tons on two axles at 10 ft. centres and 6 ft. gauge, two-thirds of the load being carried on the rear axle, as shown in Fig. 1.

Wind load on loaded chord, 300 lb. per lineal ft. and on unloaded chord 150 lb. per lineal ft., in both cases to be considered as a moving load.

Permissible Stresses, etc. In accordance with General Specifications for Steel Highway Bridges, Department of Public Highways, Ontario, 1917.

Impact. For stringers and floor beams, 30% of maximum computed live load stress; for all other members, except those subjected to alternate stresses, 10% of the live load stress.

Minimum Sections. Material, except in lattice bars and fillers, not less than 5/16 in. thick. Minimum angle, except for hand rails, 2½ × 2½ × 5/16 in.

Rivets. ¾ in. dia.

Floor Slab

Spacing of stringers must not exceed 3½ ft. (See Clause 65 of specification.) For the centre to centre width of bridge assumed, five spaces of about 3 ft. each will be necessary, as seen from Fig. 1.

For Class A loading, concrete floor slabs are to be designed for a load of 3,000 lb. applied midway between stringers and resting on a base one foot square and assumed as supported by a transverse

strip of slab one foot wide, as shown in Fig. 2. Simple support is assumed, as the restraint ordinarily obtained at supports is uncertain.

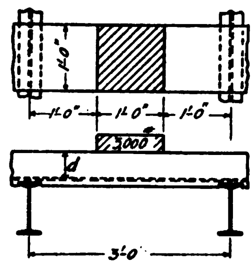


FIG. 2—LOAD ON ONE-FT. STRIP OF FLOOR SLAB

Moment on 1-ft. Strip of Slab.

Assume a 6-in slab, weighing 75 lb. per sq. ft. of area. Neglecting end restraint, therefore,

D. L. M. = $\frac{1}{8} \times 75 \times (3)^2$	=	85 ft.-lb.
L. L. M. = 1500 (1.5 — 0.25)	=	1,875 ft.-lb.
Imp. = 30% of 1,875	=	563 ft. lb.
		2,523

or 2,523 × 12 = 30,275 in.-lb.

Depth and Reinforcement of Slab.

On the basis of a permissible flexural stress in concrete, f_c, = 650 lb. per sq. in., and a permissible tensile stress in steel, f_s, = 16,000 lb. per sq. in., the depth of the slab to centre of steel, d, must be at least

$$(M/1296)^{1/2} = (30{,}275/1296)^{1/2} = 4.82 \text{ in.}$$

Adding $1\frac{1}{8}$ in. for the distance between centre of steel and bottom of slab, the total thickness required is 5.95, say 6 in.

Area of reinforcement per foot of width of slab = $A_s = M/f_s jd$. If j be taken as 7/8, and assuming that the actual depth to centre of steel = 6.00 — 1.13 = 4.87 in., A_s = 30,275/16,000 × 0.875 × 4.87 = 0.44 sq. in. This may be supplied in the form of a mesh material or by rods. If the latter type is used, a number of rods, giving an area of about 25% of that supplied transverse to the stringers must be run longitudinally, that is parallel to the stringers, to provide for temperature and shrinkage stresses.

MAIN STRINGERS

According to the specification, when stringers are not more than 3 ft. centre to centre, or when a reinforced concrete floor is used, one-half of the concentrated wheel load shall be considered as carried by one stringer.

For a 15-ton concentrated load on four wheels, arranged as shown in Fig. 1, the conditions for maximum moment on a 12-ft. stringer require the placing of one of the rear wheels at the centre of the stringer. One-half of a rear wheel load, or $0.5 \times 10{,}000 = 5{,}000$ lb., is therefore the single concentrated load at the middle. Assuming a weight of stringer of, say, 25 lb. per ft., the dead weight per lineal foot carried by a stringer is (1) concrete, $3 \times 75 = 225$ lb., and (2) steel, 25 lb., making a total of 250 lb. per lineal ft.

Maximum Moment and Stringer Section.

D. L. M. $= \frac{1}{8} \times 250 \times (12)^2$	=	4,500 ft.-lb.
L. L. M. $= \frac{1}{4} \times 5{,}000 \times 12$	=	15,000 ft.-lb.
Imp. = 30% of 15,000		4,500 ft.-lb.
		24,000 ft.-lb.

$= 24{,}000 \times 12 = 288{,}000$ in.-lb.

The moment due to uniform live load is much less than that due to the concentrated load, and hence the stringers must be designed for the latter loading.

Section modulus, S, required $= M/f$
$= 288{,}000/16{,}000 = 18$. Use a 9-in. I at 21.8 lb., for which $S = 18.9$.

Maximum Stringer Reaction and End Shear.

The maximum stringer reaction occurs when a rear wheel of the concentrated load is as close as possible to the end of the stringer without actually bearing on the floor beam. Allowing for the distribution of this 10,000-lb. load in such a way that 5,000 lb. of it is carried by the stringer under the load, the stringer reaction due to live load will be 5,000 lb. plus the portion of the front wheel load carried to this end. This would be $0.5 \times 5{,}000 \times 2/12 = 400$ lb., so that the total live load reaction is 5,400 lb. Adding 30% for impact and including the dead load reaction, or 1,500 lb., the total reaction of one stringer = 8,500 lb. The web of the stringer provided is ample in area to resist shearing and buckling stresses arising from this reaction.

Jack Stringers

As the outer wheel of the rear axle of the concentrated load will, when close up to the wheel guard, be practically over the jack stringer, and as the floor slab terminates laterally but slightly beyond this stringer, there will be less opportunity for distribution of a concentrated load among a number of stringers than in the case of a main stringer. Consequently a jack stringer may be called upon to bear a central concentrated load somewhat in excess of 5,000 lb. However, the dead load is less than on a main stringer, and hence it will be made of the same section as a main stringer, that is a 9-in. I at 21.8 lb. per ft.

Floor Beams

Span is assumed as distance centre to centre of trusses = 17 ft. 6 in.

Dead load consists of (1) weight of floor beam itself, which may be assumed for trial at 65 lb. per lineal ft., or in all 1,200 lb., uniformly distributed, and (2) weight of the part of the stringers and floor slab supported by one floor beam. The latter, which may be regarded as uniformly distributed over a central length of the floor-beam of 16 ft., as shown in Fig. 3 (a) consists of one panel of floor, that is 12 ft. This comprises 6 stringers weighing 21.8 lb. per lineal

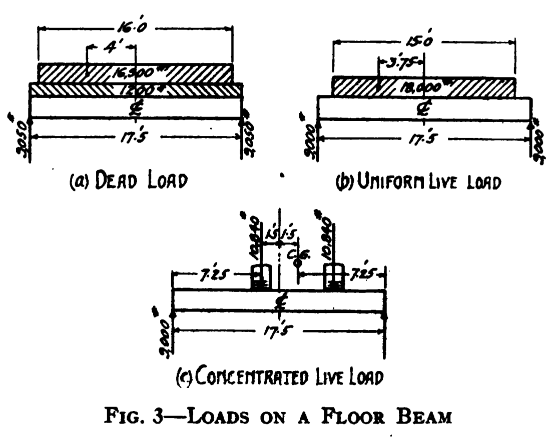


Fig. 3—Loads on a Floor Beam

ft., or 130.8 lb. for all stringers plus a total width of 17 ft. of 6-in. floor slab, (counting the two wheel guards), making a weight of concrete of 17 × 75 = 1275 lb. per lineal ft. of bridge. The dead

load from stringers and floor borne by one floor beam is, therefore = $(130.8 + 1275) \times 12 = 16,900$ lb.

Two alternative live loads must be considered, (1) the uniform live load, and (2) the concentrated load. The first may be assumed as distributed over a floor width of 15 ft., according to the specification, and consequently there would be borne by one floor beam on a central space of 15 ft., a live load of $15 \times 100 \times 12 = 18,000$ lb. as shown in Fig. 3 (b).

The second of the alternative loads, the roller or truck, will be assumed as applied to the floor beam at two points, that is under two wheels. This is justifiable, as the wheels over the floor beam transmit the loads directly to it, rather than through the stringers.

To produce the greatest moment in a floor beam that can be produced by the specified concentrated load, the rear axle must be placed over the floor beam so that one wheel is one-quarter of the gauge (centre to centre of wheels), that is 1.50 ft., from the centre line of the floor beam, as in Fig. 3 (c). At the point where each wheel rests, there will be not only the 10,000 lb. load from this wheel, but a transferred load from the corresponding wheel on the front axle amounting to 2/12 of 5,000 lb. = 840 lb., or a total load at each point of 10,840 lb. The maximum live load moment will be under the wheel which is 1.50 ft. from the centre line of the floor beam. For this arrangement, the left-hand reaction is $(21,680 \times 7.25) \div 17.5 = 9,000$ lb.

Maximum Moment and Floor Beam Section

D. L. M. = $9050 \times 8.75 - (600 \times 4.37 - 8450 \times 4)$	= 42,880 ft.-lb.
L. L. M. (concentrated) = 9000×7.25	= 65,250 ft.-lb.
Impact = 30% of 65,250	= 19,580 ft.-lb.
	127,710 ft.-lb.

$= 127,710 \times 12 = 1,530,000$ in.-lb.

The moment due to uniform live load is only $9,000 \times (8.75 - 3.75) = 45,000$ ft.-lb., and hence the design is made on the basis of the concentrated live load.

Section modulus, S, required = $M/f = 1,530,000/16,000 = 95.7$. Use a 20-in. I at 65.4 lb., for which $S = 117$.

Maximum Floor Beam Reaction and End Shear.

D. L. Reaction = ½ (16,900 + 1,200)	=	9,050 lb.
L. L. Reaction, due to concentrated load as found under "Live Load Stresses in Trusses"	=	15,400 lb.
Impact = 30% of 15,400	=	4,620 lb.
		29,100 lb.

The web is adequate to resist this reaction.

Dead Load Stresses in Trusses

Estimated dead load borne by trusses, shown in Fig. 1, will be as follows:

Steel	525 lb. per lin. ft.
Concrete	1275 lb. per lin. ft.
Total	1800 lb. per lin. ft.

Panel dead load per truss = ½ × 1800 × 12 = 10,800 lb., all of which will be assumed as applied at the bottom chord panel points.

Length of truss diagonal = $\{(6)^2 + (8)^2\}^{1/2} = 10$ ft.

Sec. $\theta = 10 / 8 = 1.25$.

Dead load reaction of one truss, if no end floor beams are used, is approximately 2 panels of dead load, or 2 × 10,800 = 21,600 lb.

Dead load stresses, calculated as below, are indicated on the stress sheet, Fig. 1.

Dead Load Shears and Web Stresses.

Panel	D. L. Shear	
ac	21,600 — 0 =	21,600 lb.
ce	21,600 — 10,800 =	10,800 lb.
ef	21,600 — 21,600 =	0

Member	D. L. Stress	
aB, *Bc*	21,600 × 1.25 =	27,000 lb.
cD, *De*	10,800 × 1.25 =	13,500 lb.
eF, *Fg*		0 lb.

Dead Load Moments and Chord Stresses.

Panel Point	D. L. Moment	
B	$21{,}600 \times 6$	= 129,600 ft.-lb.
c	$21{,}600 \times 12$	= 259,200 ft.-lb.
D	$21{,}600 \times 18 - 10{,}800 \times 6$	= 324,000 ft.-lb.
e	$21{,}600 \times 24 - 10{,}800 \times 12$	= 388,800 ft.-lb.
F	$21{,}600 \times 30 - 10{,}800\,(18 + 6)$	= 388,800 ft.-lb.

Member	D. L. Stress	
ac	$129{,}000 \div 8$	= 16,200 lb.
BD	$259{,}000 \div 8$	= 32,400 lb.
ce	$324{,}000 \div 8$	= 40,500 lb.
D	$388{,}800 \div 8$	= 48,600 lb.
F	$388{,}800 \div 8$	= 48,600 lb.
eg		

LIVE LOAD STRESSES IN TRUSSES

Panel live load per truss due to uniform live load = $\frac{1}{2} \times 1500 \times 12 = 9{,}000$ lb.

Uniform live load reaction of one truss, if no end flow beam be used, is 2 panels of live load, or $2 \times 9{,}000 = 18{,}000$ lb.

Uniform Live Load Shears and Web Stresses.

The conventional assumption respecting conditions for maximum live load shears in the various panels will be made, namely, that there is a full panel live load to the right of the panel under consideration and no panel live loads to the left.

Panel	Maximum U. L. L. Shear	
ac	$18{,}000 - 0$	= 18,000 lb.
ce	$9{,}000\,(1/5 + 2/5 + 3/5)$	= 10,800 lb.
eg	$9{,}000\,(1/5 + 2/5)$	= 5,400 lb.
gi	$9{,}000 \times 1/5$	= 1,800 lb.

Member	Maximum U. L. L. Stress	
aB, Bc	$18{,}000 \times 1.25$	= 22,500 lb.
cD, De	$10{,}800 \times 1.25$	= 13,500 lb.
eF, Fg	$5{,}400 \times 1.25$	= 6,750 lb.
gH, Hi	$1{,}800 \times 1.25$	= 2,250 lb.

Uniform Live Load Moments and Chord Stresses.

For maximum live load chord stresses due to uniform live load the bridge floor must be entirely covered with the specified uniform load of 100 lb. per sq. ft.

Panel Point	U. L. L. Moment	
B	18,000 × 6	= 108,000 ft.-lb.
c	18,000 × 12	= 216,000 ft.-lb.
D	18,000 × 18 — 9,000 × 6	= 270,000 ft.-lb.
e	18,000 × 24 — 9,000 × 12	= 324,000 ft.-lb.
F	18,000 × 30 — 9,000 (18 + 6)	= 324,000 ft.-lb.

Member	U. L. L. Stress	
ac	108,000 ÷ 8	= 13,500 lb.
BD	216,000 ÷ 8	= 27,000 lb.
ce	270,000 ÷ 8	= 33,800 lb.
DF	324,000 ÷ 8	= 40,500 lb.
eg	324,000 ÷ 8	= 40,500 lb.

Live Load Stresses Due to Concentrated Load.

The effect of the moving concentrated load on a truss will be greatest when the roller or truck is placed as close as possible to the wheel guard next the truss considered. The maximum panel point concentration due to this loading will arise when the rear axle is over the floor beam coming in at the panel point in question.

Assuming the rear wheels to have a 20-in face and to be placed tight up to the wheel guard, which is located with respect to the trusses as shown in the stress sheet, Fig. 1, the centre of the rear wheel would be about 2.1 ft. from the centre line of the near truss, as indicated in Fig. 4.

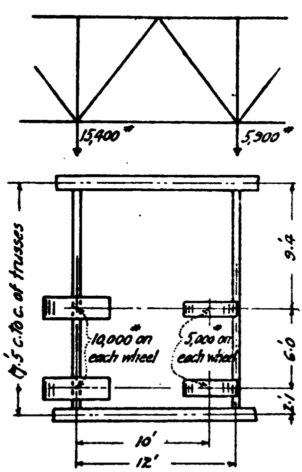


FIG. 4—MAXIMUM PANEL POINT CONCENTRATIONS PRODUCED BY CONCENTRATED LOADING

Two concentrated live loads are brought to bear on the floor beam under the heavy axle, each consisting of the 10,000-lb. load on each rear wheel, plus the portion of the front wheel load transferred to the floor beam under the rear axle, or 5,000 × 2/12 = 840 lb. At each of the rear wheels there will therefore be a concentration of 10,840 lb. At points on the floor beam which is nearest the front axle, and in the planes of the wheels on each side of the roller or truck, there will be concentrations of 5,000 × 10/12 = 4,170 lb.

The panel point concentration at the floor beam under the rear axle, therefore, is 10,840 (9.4 + 15.4) ÷ 17.5 = 15,400 lb. and at the panel point nearest the front axle it is 4170 (9.4 + 15.4) ÷ 17.5 = 5,900 lb. These loads are shown applied to the truss in Fig. 4.

The maximum positive shear in a panel will occur when the rear axle is over the panel point immediately to the right of the panel under consideration, and the front, or lightly-loaded, axle is to the right of the heavily-loaded axle. The live load shears in successive panels due to the roller or truck, and the web stresses resulting therefrom, will then be as follows:

Panel	Concentrated L. L. Shear	
ac	15,400 × 4/5 + 5,900 × 3/5	= 15,900 lb.
ce	15,400 × 3/5 + 5,900 × 2/5	= 11,600 lb.
eg	15,400 × 2/5 + 5,900 × 1/5	= 7,300 lb.
gi	15,400 × 1/5	= 3,100 lb.

Member	Concentrated L. L. Web Stresses	
aB, *Bc*	15,900 × 1.25	= 19,900 lb.
cD, *De*	11,600 × 1.25	= 14,500 lb.
eF, *Fg*	7,300 × 1.25	= 9,100 lb.
gH, *Hi*	3,100 × 1.25	= 3,900 lb.

The maximum moment at a panel point due to the roller or truck loading will arise when the 15,400-lb. concentration is at the panel point under consideration and the 5,900-lb. concentration is on the longer segment of the truss. This position will satisfy the criterion that the load per panel to the left of the point must be equal to the load per panel over the whole span, and will give a larger moment than would be given if the 5,900-lb. concentration were on the shorter segment.

Maximum Concentrated L. L. M. at Panel Points "c" and "e."

Concentration of 15,400 lb. at c and 5,900 lb. at e will give condition for maximum moment at c. Left hand reaction of truss $= 15{,}400 \times 4/5 + 5{,}900 \times 3/5 = 15{,}900$ lb. Moment at $c = 15{,}900 \times 12 = 190{,}800$ ft.-lb., which is less than the moment produced by the uniform live load.

Concentration of 15,400 lb. at e and 5,900 lb. at g will give condition for maximum moment at e. Left hand reaction of truss $= 15{,}400 \times 3/5 + 5{,}900 \times 2/5 = 11{,}600$ lb. Moment at $e = 11{,}600 \times 24 = 278{,}400$ ft.-lb., which is less than the moment produced by the uniform live load.

From the above it is seen that the maximum live load chord stresses arise from the uniform load, and not from the concentrated load.

Wind Stresses

Since no lateral bracing can be used between the unloaded chords of a pony truss span, the stresses in the one lateral truss provided, that in the plane of the bottom chords, must be computed for a moving load of 450 lb. per lineal foot.

Panel wind load $= 450 \times 12 = 5{,}400$ lb., considered as applied wholly to the windward chord.

Horizontal reaction at abutments, for purposes of calculating lateral truss stresses, when full length of bridge is subjected to wind pressure $= 5{,}400 \times 2 = 10{,}800$ lb.

Maximum stresses in lateral diagonals will arise when maximum shear occurs in panel, that is when all panel points to right of the panel considered are fully loaded and when there are no panel point loads to the left. The maximum wind load shears are therefore as follows (see Fig. 1):

Panel	Maximum Wind Load Shear	
a_1c	$10{,}800 - 0$	$= 10{,}800$ lb.
ce	$5{,}400 \times (3/5 + 2/5 + 1/5)$	$= 6{,}500$ lb.
eg	$5{,}400\ (2/5 + 1/5)$	$= 3{,}250$ lb.

Since the lateral diagonals will be single angles of slender dimensions, they are unable to take appreciable compression and will be here regarded as taking tension only. The maximum diagonal

stresses are therefore as follows, section θ' being, for regular panels, 1.21:

Member	Maximum Wind Load Diagonal Stresses	
a_1c'	10,800 × 1.21	= 13,100 lb.
ce'	6,500 × 1.21	= 7,900 lb.
eg'	3,250 × 1.21	= 3,900 lb.

The insertion of an end wind strut makes the length of the lateral truss panel a_1c somewhat less than the other panels, but the effect of the irregularity on the diagonal and chord stresses in this panel is inappreciable.

Maximum chord stresses in the lateral truss will arise when the structure is fully covered with the wind load. The wind moments at panel points and the resulting chord stresses are as indicated below:

Panel point	Maximum Wind Moment	
c, c'	10,800 × 12	= 129,600 ft.-lb.
e, e'	10,800 × 24 — 5,400 × 12	= 194,400 ft.-lb.

Member	Maximum Wind Load Stress	
$a_1c, c'e'$	129,600 ÷ 17.5	= 7,400 lb.
$ce, e'g'$	194,400 ÷ 17.5	= 11,100 lb.

The wind compression in the floor beams, which act as intermediate struts for the lateral truss, are as shown in Fig. 1. They are equal to the wind shear in the panel to the left, in each case. For the end strut a_1a', the wind thrust is computed on the assumption that one-half of the total wind reaction at the end is carried across by the strut from the windward to the leeward truss bearing. The stress in the strut is therefore one-half of 2½ panels of wind load, or 0.5 × 2.5 × 5,400 = 6,750 lb.

Combination of Truss Stresses

The dead load and maximum live load stresses found for the truss members are indicated on the truss diagram of the stress sheet, Fig. 1. The minimum live load stresses in web members arise from the concentrated loading in the case of all panels but the end ones, where the uniform live load produces larger web member stresses. The chords are everywhere stressed more highly by the uniform live load than by the concentrated load.

Impact, to the extent of 10% of the maximum computed live load stress is added to the stress in each truss member, except in the case of diagonals *eF* and *Fg*, in which reversal of stress may be brought about by the concentrated loading. For each of these diagonals, the dead load stress is zero, and the live load stress may be either a tension of 9,100 lb., or a compression of like amount. In accordance with the rule of the specifications, the impact allowance will be 50 % of the maximum live load stress, or 4,600 lb.

Since the axial wind stresses in the chords in no case exceed 25% of the sum of the dead load, live load and impact stresses, they will be neglected.

Proportioning of Truss Members

According to the specification, the permissible stress for axial tension on net section is 16,000 lb. per sq. in., and axial compression is limited to $16{,}000 - 70\ l/r$. where l = unsupported length of member in inches, and "r" the corresponding radius of gyration. The length of a compression member must not exceed 100 times its least radius of gyration for main members, nor 120 times for laterals, struts and wind bracing. Bending stresses must not exceed 16,000 lb. per sq. in.

End Post aB.

The total stress, made up as indicated on the stress sheet, Fig. 1, is — 51,800 lb.

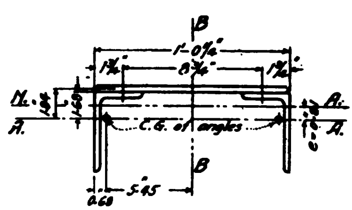


Fig. 5—Section of End Post and Top Chord

Assume, as section, two angles, 5 × 3 × 5/16 in., 12¼ in. back to back, and a cover plate 12 × 5/16 in., arranged as shown in Fig. 5. The distance between rivet lines in the cover plate is 8.75 / 0.3125 = 28 times the thickness of the plate, which is well within the limit of 40 set by the specification. A 12-in. plate is chosen to give considerable lateral rigidity to the truss. Material less than 5/16 in. must not be used.

To find the neutral axis of the section, take moments about the gravity axis AA of the angles, which is 1.68 in. from the back of the shorter legs.

Part	Area	Arm	Statical Moment
2 angles	4.80	0	0
1 plate	3.75	+ 1.84	+ 6.90
	8.55		+ 6.90

Distance, e, of neutral axis of section from axis AA, the centre of gravity of the angles, = + 6.90 / 8.55 = + 0.81 in.

Moment of inertia of section about $NA = I_{NA} = I_{AA} - Ae^2 =$ {moment of inertia of two angles about their own gravity axis + moment of inertia of plate about its own gravity axis (a negligible quantity) + area of plate multiplied by the square of the distance between its gravity axis and the axis AA} — area of whole section multiplied by the square of the distance of its neutral axis from the axis AA. This in figures $= \{2\ (6.26) + 0 + 12 \times 0.3125 \times (1.84)^2\} - 8.55 \times (0.81)^2 = 19.62$ in.^{4}

Radius of gyration about axis $NA = (19.62 / 8.55)^{1/2} = 1.51$ in.

Similarly, the moment of inertia about axis BB is found to be 190.2 in.^{4} and the radius of gyration is found to be 4.71 in.

The maximum slenderness ratio, l/r, hence occurs in the plane of the truss and = 120 / 1.51 = 80.

Permissible compressive stress $= p = 16{,}000 - 70\ l/r = 16{,}000 - 70 \times 80 = 10{,}400$ lb. per sq. in.

Area required = 51,800 / 10,400 = 4.98 sq. in. The area provided is 8.55 sq. in., but this cannot be reduced without violating the provisions of the specification respecting minimum sections and maximum slenderness ratio.

Top Chord BCD.

Total stress = — 62,100 lb.

Assuming the same section as for aB, the slenderness ratio in the plane of the truss, considering the chord to be supported in a vertical place at B, C and D, is 72 / 1.51 = 48. In a plane at right angles at the plane of the truss, support is afforded by the inclined braces at C and E (see Fig. 1) but that furnished at D and F is

negligible. Consequently, l/r at right angles to the plane of the truss = 144 / 4.71 = 31.

Permissible compressive stress = p = 16,000 — 70 × 48 = 12,640 lb. per sq. in.

Area required = 62,100 / 12,640 = 4.92 sq. in. For the sake of the appearance, and to simplify details, the same section as was used for aB will be employed.

Top Chord DEF.

Total stress = — 93,200 lb.

As will be seen from the stress sheet, Fig. 1, the same section as was used for aB and BD will be employed.

Bottom Chord ac or a′c′.

This member must be proportioned to withstand both axial stress and the bending due to the transverse wind force of 6750 lb. carried in to it by the wind strut $a_{,}a'_{,}$ at a point about 15 in. from the centre of the end bearing of the truss. Since from the nature of the lateral truss, there can be no axial wind stress in the leeward chord of the end panel, although there is wind compression in the windward chord, the leeward chord member $a'c'$ is more severely stressed than ac. This member carries an axial tension of 31,100 lb., and a wind moment at the wind strut connection of (6,750 × 10.75/12) × 15 = 91,600 in. lb.

Consider, first, the dead load, live load and impact stresses only. The area required = 31,100 / 16,000 = 1.95 sq. in. Two angles, 3 × 2½ × 5/16 in. will be assumed with the 3-in. legs vertical and with battens or tie plates on the 2½-in. legs. Allowing for a gauge line in each leg of each angle, the normal distance between these lines of rivets, if the angle were developed, would be 2 13/16 in. If the pitch of rivets at the ends were 3 in. in each leg—a probable value—then for the 1½-in. stagger, the diagonal distance between rivets would be 3 3/16 in. Since this is less than 40% in excess of the distance between gauge lines, two rivet holes will need to be deducted from each angle, according to the specification. The net area of the section assumed is, therefore, 3.26 — 4 × 0.875 × 0.3125 = 2.16 sq. in. and the section is adequate.

Considering the combination of axial stress and wind moment, the stress may be allowed to attain 20,000 lb. per sq. in., according

to the specification. As the wind strut a,a', connects to the chord near the edge of the gusset plate, it is best to deduct two holes from each angle at the wind strut connection. The gross moment of inertia of the two angles about a vertical axis is 98, the net moment of inertia, assuming that it is reduced by the holes in approximately the same ratio as the area, is about 65, and the corresponding radius of gyration is 5.48 in. The net area required in the member is, therefore, $P/p + My_1/r^2p = 31{,}100 / 20{,}000 + 91{,}600 \times 6.125 / \{(5.48)^2 \times 20{,}000\} = 1.55 + 0.93 = 2.48$ sq. in. This is in excess of the net area of the section assumed and hence it must be increased.

Assuming two $3\frac{1}{2} \times 3 \times 5/16$-in. angles, with a net area, allowing for four holes out, of 2.76 sq. in. net, a net moment of inertia transverse to the truss of approximately 80, and a corresponding radius of gyration of 5.37 in., the net area required $= 31{,}100 / 20{,}000 + 91{,}600 \times 6.125 / \{(5.37)^2 \times 20{,}000\} = 1.55 + 0.97 = 2.52$ sq. in. This section is adequate.

Bottom Chord ce.

Total stress = + 77,700 lb.

Area required = 77,700 / 16,000 = 4.85 sq. in. Material provided = 2 angles, $5 \times 3\frac{1}{2} \times 3/8$ in. = 6.10 sq. in., gross. Deducting four rivet holes, the net area = 4.79 sq. in. which is sufficiently near the requirement.

Bottom Chord eg.

Total stress = + 93,200 lb.

Area required = 93,200 / 16,000 = 5.83 sq. in. Two $5 \times 3\frac{1}{2} \times \frac{1}{2}$-in. Angles are provided which, with four holes deducted, give 6.25 sq. in. net.

Diagonal Bc.

Total stress = + 51,800 lb. Material provided, as per stress sheet, is 2 angles, $4 \times 3 \times 3/8$ in.

Diagonal cD.

The maximum live load stress attainable in this member arises from the concentrated load, and may be — 14,500 lb. or + 3,900 lb. As the live load tension can never offset the live load compression, the impact for this member is 10% of the maximum compressive

live load stress, and the stress for which the member must be designed is — 29,500 lb. As will be seen from the stress sheet, two angles, 4 × 3 × 5/16 in. will be adequate.

Diagonal De.

The maximum stress in this member is the same as for *cD* but it is of opposite sign, namely, tension. Two angles, 3 × 2½ × 5/16 in. are adequate.

Diagonal eF.

In this diagonal the dead load stress is zero and the maximum live load stress is + 9,100 lb. or — 9,100 lb. During the passage of the concentrated load across the bridge in one direction, however, from left to right, as shown in Fig. 6, the stress in *eF* would vary between + 6,900 for position I and — 9,100 lb. for position II, and, during the passage of the load from right to left it would vary from—6,900 lb. to + 9,100 lb. This member is, therefore, subjected to alternate or reversed stress, and for it an impact allowance of 50% of the greater live load stress is required by the specification. The total maximum stress in the member may therefore be + 13,700 or — 13,700 lb. For this, two angles 4 × 3 × 5/16 in. will need to be employed to keep the slenderness ratio below 100, although lighter angles would carry the stress safely.

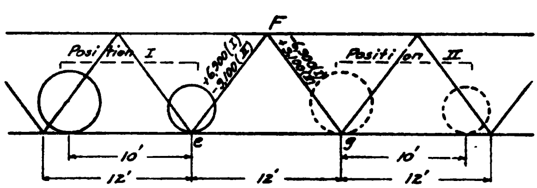


FIG. 6—POSITIONS OF CONCENTRATED LOADING PRODUCING MAXIMUM STRESSES IN DIAGONALS OF CENTRE PANEL

Verticals Cc, Ee, etc.

As these serve only to shorten the unsupported length of the top chord sections and to afford a place of attachment for the handrail, they will be made of the minimum sections practicable for satisfactory details. The verticals will consist of two angles, 3½ × 2½ ×5/16 in, with the 3½-in. legs in the place of the truss, and the batter brace of a single angle of the same size with the 3½-in. leg at right angles to the transverse bracing.

Lateral System.

The maximum stress in an end diagonal is 13,100 lb. Area required = 13,100 / 16,000 = 0.82 sq. in. One angle $2\frac{1}{2} \times 2\frac{1}{2} \times 5/16$ in. with one hole deducted gives 1.20 sq. in. net, and hence is adequate. The same size will be used for all diagonals, as this is the minimum section permitted.

The compressive wind stresses in the floor beams, which act as intermediate wind struts, are negligible in comparison with the heavy flexual stresses existing therein. As no wind floor beam is used, an angle strut will be inserted to carry the computed wind compression of 6,750 lb. This strut will be attached to the bottom flanges of the end stringers by a special detail, and so is supported vertically every 3 ft. Horizontally, its unsupported length is approximately 200 in. A $5 \times 3\frac{1}{2} \times 5/16$-in. angle with the 5-in. leg placed horizontally will give a value of l/r of 200 / 1.61 = 124. This is sufficiently near the limit 120, considering the large amount of excess area provided for the calculated stress.

End Channel.

To serve as a finish for the concrete floor and to carry the checker plate provided to bridge the gap between the floor and the ballast wall of the abutment, a 6-in. channel at 10.5 lb. is run across the tops of the end stringers and connected to the end posts of the handrail.

Handrail.

The handrail will be of the latticed type, made up as shown in the stress sheet, Fig. 1, and in the details, Fig. 7. The top of the upper rail must not be less than 4 ft. above the level of the concrete floor and this rail must withstand a horizontal force of 60 lb. per lineal ft. For a span of 12 ft. the lateral moment is $\frac{1}{8} \times 60 \times (12)^2 \times 12 = 12{,}960$ in.-lb. Section modulus required = 12,960 / 16,000 = 0.81. Section modulus of a $4 \times 2\frac{1}{2} \times 5/16$-in. T is 0.88, and hence this is adequate.

Details

General.

Truss members will be connected so that the centre of gravity of the group of rivets employed in end connections will lie as close as practicable to the skeleton line of the truss, as will be seen from

the general details, Fig. 7. Thus, the back of the main angles, for diagonal Bc is placed on the skeleton line. While the total gross area of both legs of an angle may be considered as effective for carrying compressive stress, whether one or both legs be connected, only the connected leg of an angle which is connected by one leg only may be counted. (Para. 17, specification.) To cheapen and simplify construction, rivet spacing will be made as close as practicable and gusset plates as small as possible. Plates should, in general, be rectangular and of such widths as are available in the open market.

Permissible shearing and bearing stresses on shop rivets are 10,000 and 20,000 lb. per sq. in., and on field rivets 8,000 and 16,000 lb. per sq. in., respectively. Rivets, in general, will be ¾ in. dia., except 5/8 in. rivets in the flanges of 6-in. channels and ½ in. rivets in the lighter material of the handrail.

Body Details.

The unsupported sides of all compression members will be stayed by battens at distances apart not exceeding 3 ft. For end posts and top chord, these battens take the form of a 6 × 5/16-in. plate with two 3½ × 3 × 5/16-in. connection angles. For the compression diagonals, 6 × 5/16-in. tie plates with two rivets in each end will suffice. Cover plates of end posts and top chords are riveted to the angles by two lines of rivets at the maximum permissible spacing of 16 × 5/16 = 5 in., except at the ends, where closer spacing is used for a distance of about twice the width of the member.

Tension members composed of two angles will be tied together by 6 × 5/16-in. tie plates at intervals of not over 4 ft., in order to stiffen the members and make the component parts act as a unit.

The three angles of each vertical are tied together, as shown in Fig. 7, by 6 × 5/16-in. horizontal plates.

Joint a.

Corrections will be made at all joints by two 5/16-in. gusset plates placed on the outside of the members joined. The least value of rivets through the gusset plates would not be raised by increasing the thickness of the plates, as this value is in single shear. This is 0.442 × 10,000 = 4,420 lb. for shop rivets.

Maximum stress in end post aB = 51,800 lb. Number of rivets required = 51,800 / 4,420 = 12. This number is provided.

Maximum axial stress in bottom chord ac = 31,100 lb. Number of rivets required = 31,100 / 4420 = 7. Eight rivets are provided, not counting those required for the attachment of the handrail post to the truss.

The gussets are chipped to bear on the shoe plate so as to overcome the necessity of inserting extra rivets to take the vertical component of the end post stress into the bottom chord angles.

Rectangular gusset plates, 20 × 5/16 in. may be used.

Joint B.

As the bent plate at the hip is not capable of transmitting appreciable stress across the joint, the numbers of rivets required and provided in the ends of the three members meeting at the joint to deliver to the gussets the maximum stresses indicated on the stress sheet, Fig. 1, are as follows:

aB	51,800 / 4420 = 12 shop rivets; 12 provided.
BD	62,100 / 4420 = 14 shop rivets; 14 provided.
Bc	51,800 / 4420 = 12 shop rivets; 12 provided.

To render both legs of the tension diagonal Bc effective, lug angles, 3 × 3 × 5/16 in. are employed, with three rivets in each leg.

Gusset plates 18 × 5/16 in. with parallel edges are adopted.

Joints C and E.

Since no calculable stress is carried by the vertical Cc, two rivets in the end of each angle will be sufficient. The gusset plates are set back from the backs of the vertical angles to permit the attachment of a transverse connecting plate for the outside batter brace. Joint E is the same as C.

Joint D.

About 3½ in. outside this point, in the member BD, a splice will be made in the top chord, for although the same material is used throughout the chord, considerations of shipping and erection would ordinarily make it desirable to handle the truss in three main sections. The gusset plates will be shop riveted to the chord section DF and De and field riveted to BD and cD. To distribute the field riveting and to facilitate ease of erection, the 12 × 3/8 in. cover splice plate will be shop riveted to BD and field riveted to DF. In accordance with para. 29 of the specification, the faced abutting

ends of the top chord at this joint will be assumed to transmit 20% of the stress in the member BD, so that the riveting will need to develop only $0.80 \times 62{,}100 = 49{,}700$ lb. The $19 \times 5/16$-in. gusset plates will act as splice plates for the greater part of the area of the angles, and the $12 \times 3/8$-in. splice plate will serve to splice the cover plate and contribute also to the splice of the angles.

Least value of all rivets is in single shear, being for shop rivets 4420 lb., and for field rivets $0.442 \times 8{,}000 = 3{,}540$ lb. The numbers of rivets required and provided in the diagonals cD and De are as follows:

cD	29,500 / 3540 =	9 field rivets; 10 provided.
De	29,500 / 4420 =	7 shop rivets; 8 provided.

To transmit 49,700 lb. from BD across the splice, 8 field rivets in the vertical legs of the chord angles and 7 shop rivets through the horizontal legs are employed, developing $8 \times 3540 + 7 \times 4420 = 59{,}300$ lb.

The total stress in DF, or 93,200 lb., will be transferred to the joint by 16 shop rivets and 9 field rivets, arranged as shown in Fig. 7, and capable of developing $16 \times 4420 + 9 \times 3540 = 102{,}400$ lb.

Joint F.

Least value of shop rivets = 4,420 lb. and hence 6 rivets in each diagonal will suffice. The rivets in the chord need be sufficient only to transmit to the chord the maximum difference of the horizontal components of the live load stresses in eF and Fg. In this, 10 rivets are ample, as this number would more than develop the maximum stress in either diagonal.

Joint c.

A shop splice in the bottom chord section ac near this joint is necessary, because of the change of section at the joint. All rivets are shop rivets and their least value is 4,420 lb. The numbers of rivets required and provided in the truss members are as follows:

ac	31,100 / 4420 =	7 shop rivets; 8 provided.
Bc	51,800 / 4420 =	12 shop rivets; 12 provided.
cD	29,500 / 4420 =	7 shop rivets; 8 provided.
ce	77,700 / 4420 =	18 shop rivets; 18 provided.

The two rivets passing through the floor beam connection angles and the angles of the chord section *ce* are not counted as available for the connection of *ce* as they are required for the purpose of delivering in part the reaction of the floor beam to the gusset plate.

A splice plate is provided on the horizontal legs of the chord angles wide enough to act also as a lateral connection plate.

The floor beam connection and diaphragm will be discussed elsewhere.

Joint e.

A field splice is necessary at this joint and is placed about 6½ in. outside the point *e* in the chord section *ce*. The splice material consists of the vertical 5/16-in. gussets, the horizontal 5/16-in. combined splice and lateral plate, two 4¼ × 3/8-in. flats on the vertical legs of the chord angles and two 2½ × 3/8-in. flats on the horizontal legs of these angles. These flats not only provide splicing material to replace the main material cut, but incidentally increase the least value of the rivets passing through them by making it the bearing value on 5/16-in., 3/8 in., and ½-in. material.

The connection of the diagonals *De* and *eF* will be the same as at their upper ends. The 77,700 lb. stress in *ce* will be transmitted across the joint by 12 field rivets in the vertical legs of the chord angles in bearing on the 3/8-in. angles, plus 4 shop rivets through the horizontal legs of these angles, in bearing on their thickness, 3/8 in., plus 2 field rivets bearing also on 3/8-in material. The latter rivets, being next the joint, are made field to permit springing the splice plates apart so that the angles of *eg* may be readily entered. The rivets provided will develop a strength of $12 \times (\frac{3}{4} \times 3/8 \times 16{,}000) + 4 \times (\frac{3}{4} \times 3/8 \times 20{,}000) + 2 \times (\frac{3}{4} \times 3/8 \times 16{,}000) = 12 \times 4500 + 4 \times 5625 + 2 \times 4500 = 85{,}500$ lb.

Bracing Connections

Three field rivets in single shear will serve for all lateral diagonal connections except those in the end panel. Here four are required, and hence lug angles are used.

Three field rivets are ample for the connection of the end wind strut.

The riveting in the outside bracing of the top chord is fixed largely by judgment, as the stresses are indeterminate.

Floor Beam Connections

The maximum reaction of a floor beam has already been found to be 29,100 lb. For this two $3\frac{1}{2} \times 3\frac{1}{2} \times 3/8$-in. angles with rivets as shown will be ample, both for the field rivets passing through the joint gusset and the shop rivets passing through the web of the floor beam. Turning effect on the latter rivets is considered.

A heavy diaphragm is provided to transfer half of the floor beam reaction from the inside to the outside gusset at the joint.

Stringer Connections

The monolithic character of the concrete floor slab admits of shelf connections of the stringers to the floor beams. These are made of heavy material, $6 \times 4 \times \frac{1}{2}$ in., to guard against bending down of the outstanding legs.

Bearings

The load transmitted to the masonry by the end bearings of a truss is, for a truss span without an end floor beam, very nearly the same as the reaction used in the computation of truss member stresses, that is, 41,400 lb. The area required for bearing on the concrete abutment at 500 lb. per sq. in. = 41,400 / 500 = 83 sq. in. Bearing and bed plates are each made 12×18 in. for reasons of detail.

DESIGN OF A SINGLE-TRACK THROUGH PRATT TRUSS RIVETED RAILWAY SPAN ON TANGENT.

DATA

Span. 175 ft. centre to centre of bearings; 7 panels of 25 ft. each.

Depth. 31 ft. c. to c. of chords.

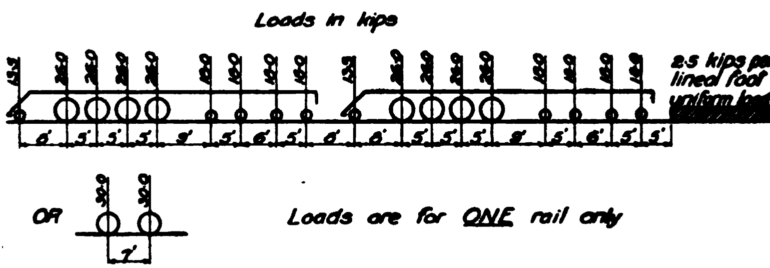


FIG. 11—CLASS "ESPECIAL HEAVY" LOADING, DOMINION GOVERNMENT SPECIFICATION, 1908.

Width. 18 ft. c. to c. of trusses; 16 ft. clear.

Stringers. 2 lines, 8 ft. c. to c.

Live Load. Class "Especial Heavy," or 120,000 lb. on two axles 7 ft. apart, as shown in Fig. 11.

Specification. General Specifications for Steel Superstructures of Bridges and Viaducts, Department of Railways and Canals, Canada, 1908.

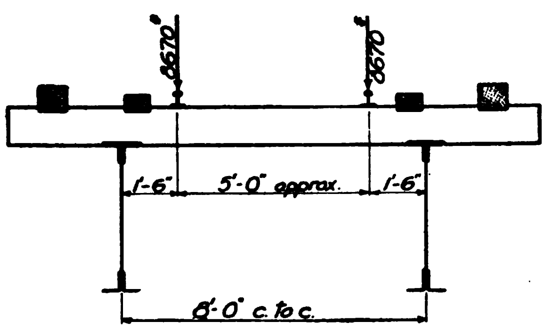


FIG. 12—MAXIMUM LOADING ON CROSS-TIES

CROSS TIES

Span c. to c. of stringers = 8 ft.

Dead load of tie is negligible compared with live load on it.

Live load per tie from each rail, due to class "Especial Heavy" loading = 26,000/3 = 8,670 lb, Fig. 12. Alternative load not considered in designing ties.

Moment on One Tie.

D. L. M., say	=	0
L. L. M. = 8,670 × 1.5 (1.40 — 0/200)	=	18,200 ft.-lb.
Imp. = $(18{,}200)^2/(18{,}200 + 0)$	=	18,200
		36,400 ft.-lb.
	=	436,800 in.-lb.

S = 436,800 / 1,800 = 243

Use 10 × 12-in. long-leaf yellow pine ties, spaced 14 in. centres. S = 240.

The section chosen is adequate for longitudinal shear.

Stringers

Span = 25 ft. c. to c. of floor beams.

Weight of timber deck fixed by specification as 600 lb. per

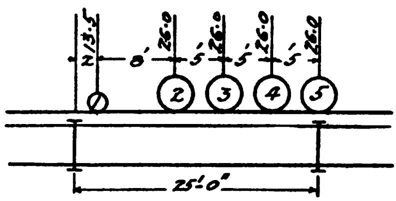


Fig. 13—Position of Live Load for Maximum End Shear on Stringer

lineal ft. for 8 × 10-in. ties. For 10 × 12-in. ties, this becomes 760 lb. per ft.

Dead Load on One Stringer.

Deck = 760 / 2	= 380 lb. per lin. ft.
Steel, in 1 stringer and half of bracing	= 240 lb. per lin. ft.
Total	620 lb. per lin. ft.

Maximum End Shear and Web Section.

For the maximum live-load end shear, place wheel 5 of the loading at one support, as shown in Fig. 13. Alternative live load gives a smaller shear than class "Especial Heavy."

Using the moment table for "Class I" loading, Fig. 9, and multiplying all loads and moments by 1.111 to conform to class "Especial Heavy."

$V_l = R_l = (981.45 / 25) \times 1.111 = 43.6$ kips
V_l = Load on first five wheels less 43.6 = kips 74 kips
D. L. V. = 620 × 25 / 2 = 7,700 lb.
L. L. V. = 74,000 × (1.40 — 25 / 200) = 94,000
Imp. = $(94{,}000)^2$ / (94,000 + 7,700) = 87,000

Total = 188,700 lb.

Web section required = 188,700 / 10,000 = 18.87 sq. in.. gross.
Use 46 × 7/17-in. plate = 20.13 sq. in.

Maximum Moment and Flange Section.

The maximum live load moment arises when the loading is placed on the stringer as shown in Fig. 14, and occurs under the wheel

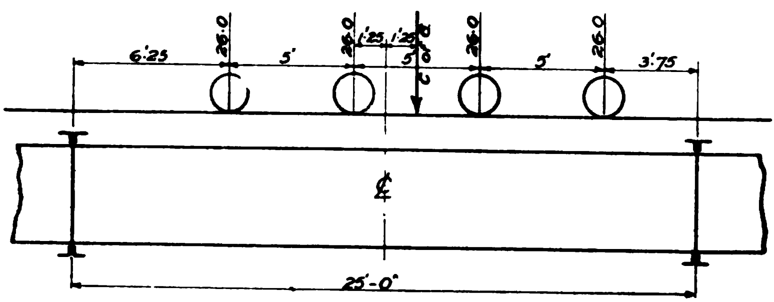


FIG. 14—POSITION OF LIVE LOAD FOR MAXIMUM MOMENT ON STRINGER

nearest the centre. Alternative live-load gives a relatively small amount.

$R_l = \{26.0\ (3.75 + 8.75 + 13.75 + 18.75)\} / 25 = 46.8$ kips
$M = 46.8 \times 11.25 - 26.0 \times 5$ = 397,500 ft.-lb.
D. L. M. = ⅛ × 620 × (25)ı = 48,400 ft.-lb.
L. L. M. = 397,500 × (1.40 — 25 / 200) = 505,000
Imp. = $(505{,}000)^2$ / (505,000 + 48,400) = 461,000

1,014,400 ft.-lb

Assuming effective depth = 46 — 2 × 1.78 = 42.44 in.
$A_f + \frac{1}{8}A_w = M / fd$ = 12,172,800 / (16,000 × 42.44) = 17.95 sq. in.

Area provided =
⅛ of web = ⅛ × 46 × 7/16 = 2.52 sq. in. gross.
2 Angles, 6 × 6 × ¾in.; less two 1-in holes = 15.38 sq. in. net

17.90 sq. in. net.

Intermediate Floor Beams

Span, centre to centre of trusses = 18 ft.

Dead load on floorbeam is assumed as 250 lb. per lineal ft.

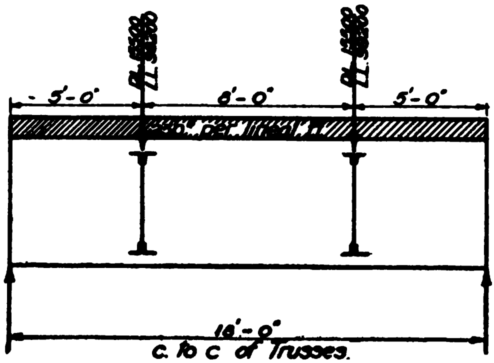


Fig. 15.—Maximum Loading on Intermediate Floor Beams

plus two dead load concentrations from the stringers, each = 620 × 25 = 15,500 lb.

The maximum live load concentrations on the floor beam arise with wheel 4 over the floorbeam, as shown in Fig. 16, since for this position $G_l = G_r$.

Reaction of loads 1, 2, and 3 at n		
$= \{13.5 \times 7 + 26\,(15 + 20)\} / 25$	=	40.2 kips
Reaction of loads 5, 6, and 7 at n		
$= \{18\,(6 + 11) + 26 \times 20\} / 25$	=	33.0
Load 4 at n	=	26.00
Total live load concentration at n	=	99.2 kips

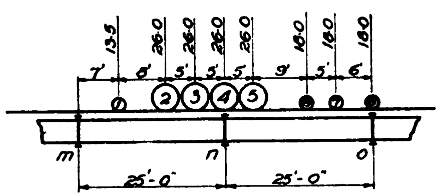


Fig. 16—Position of Live Load for Maximum Floor-beam Concentration

Maximum End Shear and Web Section.

D. L. V. = (250 × 18 / 2) + 15500	=	17,750 lb.
L. L. V. = 99,200 × (1.40 — 43 / 200)	=	118,000
Imp. = $(118{,}000)^2$ / (118,000 + 17,750)	=	102,600
		238,350 lb.

Web section required = 238,350 / 10,000
= 23.84 sq. in. gross.
Use 58 × 7/16-in. plate = 25.38 sq. in.

Maximum Moment and Flange Section.

D. L. M.

Uniform load = $\frac{1}{8}$ × 250 × $(18)^2$ = 10,100
Conc. loads = 15,500 × 5 = 77,500
87,600 ft.-lb.

L. L. M. = 99,200 × 5 (1.40 — 43 / 200) = 590,000 ft.-lb.
Imp. = $(590,000)^2$/ (590,000 + 87,600) = 514,000 ft.-lb.

1,191,600 ft.-lb.

= 14,299,200 in-lb.

Assuming for effective depth 58.5 — 2 × 1.75 = 55.0 in.
$A_f + \frac{1}{8} A_w$ = 14,299,200 / (16,000 × 55)
= 16.20 sq. in.

Area provided =
$\frac{1}{8}$ of web = $\frac{1}{8}$ × 58 × 7/16 = 3.17 sq. in.
2 angles, 6 × 6 × 11/16 in.; less two 1-in holes = 14.18 sq. in. net

17.35 sq. in. net

End Floor Beams

Dead load assumed as 220 lb. per lineal ft., plus two dead load

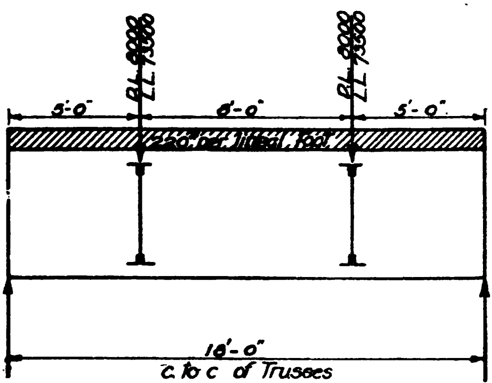


Fig. 17—Maximum Loading on End Floor Beam

concentrations from the end stringers and stringer brackets.

Stringer reaction = $650 \times 25/2$	= 7700 lb.
Stringer bracket, assumed	= 300 lb.
Total	= 8000 lb.

Maximum live load concentrations at stringer corrections arise when end shear on stringers is a maximum, and is shown in Fig. 17 as 73,500 lb. This arises with wheel 2 at the end of a stringer. Wheel 5 will produce a shear of 74,000 lb.

Maximum End Shear and Web Section.

D. L. V. = $(220 \times 18/2) + 8000$	= 9,980 lb.
L. L. V = $73{,}500 \times (1.40 - 25/200)$	= 93,400
Imp. = $(93{,}400)^2 / (93{,}400 + 9{,}980)$	= 84,300
	187,680 lb.

Web section required = 187,680 / 10,000 = 18.77 sq. in. gross.
Use $58 \times 3/8$ in. plate = 21.75 sq. in.

Maximum Moment and Flange Section.

D. L. M.

Uniform load = $\frac{1}{8} \times 220 \times (18)^2$	= 8,900
Conc. loads = $8{,}000 \times 5$	= 40,000
	48,900 ft.-lb.
L. L. M. = $73{,}500 \times 5\,(1.40 - 25/200)$	= 467,000
Imp. = $(467{,}000)^2 / (467{,}000 + 48{,}900)$	= 422,000
	937,900 ft.-lb.
	= 11,254,800 in.-lb.

Assuming for effective depth $58.5 - 2 \times 1.68 = 55.14$ in.

$A_f + \frac{1}{8} A_w = 11{,}254{,}800 / (16{,}000 \times 55.14) = 12.75$ sq. in.

Area provided =

$\frac{1}{8}$ of web = $\frac{1}{8} \times 58 \times 3/8$	= 2.72 sq. in. gross
2 angles, $6 \times 6 \times \frac{1}{2}$ in.; less two 1-in. holes	= 10.50 sq. in. net
	13.22 sq. in. net

Dead Load Stresses in Trusses

Dead Load of Span.

Steel, $w = 10\,l + 900$	= 2650 lb. per lin. ft.
Deck	= 760 lb. per lin. ft.
Total	3410 lb. per lin. ft.

Panel dead load per truss = (3410/2) × 25 = 42,600 lb.
Load at each bottom chord panel point = 2/3 × 42,600 = 28,400 lb.
Load at each top chord panel point = 1/3 × 42,600 = 14,200 lb.
Length of diagonal = $\{(25)^2 + (31)^2\}^{1/2} = 39.8$ ft.
Sec. θ = 39.8 / 31 = 1.284.
Total dead load reaction = 3½ panel loads
= 3½ × 42,600 = 149,100 lb.
For truss stresses, dead load reaction
= 3 panel loads = 3 × 42,600 = 127,800 lb.

For diagram of truss, see stress sheet, Fig. 10.

Table of Dead Load Stresses

Shear in panel *ab*	= 127,800 — 0	= 127,800 lb.
Shear in panel *bc*	= 127,800 — 42,600	= 85,200 lb.
Shear in panel *cd*	= 127,800 — (2 × 42,600)	= 42,600 lb.
Shear in panel *de*	= 127,800 — (3 × 42,600)	= 0 lb.

Moment at *B*	= 127,800 × 25	= 3,195,000 ft.-lb.
Moment at *C or c*	= 127,800 × 50 — (42,600 × 25)	= 5,325,000 ft.-lb.
Moment at *D or d*	= 127,800 × 75 — {42,600 × (25 + 50)}	= 6,390,000 ft.-lb.

Stress in member *aB*	= 127,800 × 1.284	= 164,200 lb.
Stress in member *Bc*	= 85,200 × 1.284	= 109,400 lb.
Stress in member *Cd*	= 42,600 × 1.284	= 54,700 lb.
Stress in member *De*	= 0 × 1.284	= 0 lb.
Stress in member *Bb*	= 28,400 × 1	= 28,400 lb.
Stress in member *Cc*	= 42,600 + 14,200	= 56,800 lb.
Stress in member *Dd*	= 0 × 14,200	= 14,200 lb.
Stress in member *cD*	=	= 0 lb.

Stress in member *abc*	= 3,195,000 / 31	= 103,000 lb.
Stress in member *BC* or *cd*	= 5,325,000 / 31	= 171,800 lb.
Stress in member *CDE* or *de*	= 6,390,000 / 31	= 206,000 lb.

Live Load Moments and Shears in One Truss

Use moment table for "Class I" loading, Dominion Government Specification, Fig. 9, multiplying the resulting moments or shears by 1.111 to conform to class "Especial Heavy."

Maximum Moment at "b."

Wheel 4 at b, as shown in Fig. 18.

$G/n = 473.85 \ / \ 7 = 67.69$

$G_1/m = 58.95 \ / \ 1 = 58.95$ or $82.35 \ / \ 1 = 82.35$

This position, therefore, gives a maximum moment at b. Wheels

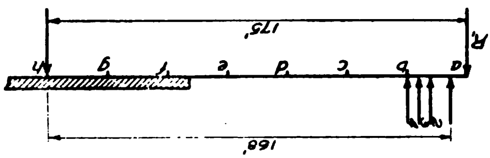


Fig. 18—Position of Live Load for Maximum Moment at Panel-point b

3 and 5 do not give the conditions for a maximum.

Using moment table, Fig. 9.

$R_1 = 43{,}585 \ / \ 175 = 249$ kips.

$M = 249 \times 25 - 570 = 5655$ kip-ft. for Class "I."

$5655 \times 1.111 = 6283$ kip-ft. for Class "Especial Heavy."

Maximum Moment at "c."

Wheel 7 at c

$G/n = 460.35 \ / \ 7 = 65.76$

$G_1/m = 121.95 \ / \ 2 = 60.98$ or $138.15 \ / \ 2 = 69.08$

Wheel 7, therefore, gives a maximum moment, but wheels 6, 8 and 9 do not.

$R_1 = 40{,}782 \ / \ 175 = 233$ kips.

$M = 233 \times 50 - 2543 = 9107$ kip-ft. for Class "I."

$= 9107 \times 1.111 = 10{,}119$ kip-ft. for Class "Especial Heavy."

Maximum Moment at "d."

Wheel 11 at d

$G/n = 464.85 \ / \ 7 = 66.41.$

$G_1/m = 182.7 \ / \ 3 = 60.9$ or $206.1 \ / \ 3 = 68.7$

Wheel 11, therefore, gives a maximum, but wheels 10 and 12 do not.

$R_l = 41{,}707 \ / \ 175 = 238.5$ kips
$M = 238.5 \times 75 - 6970 = 10{,}930$ kip-ft. for Class "I"
$= 10{,}930 \times 1.111 = 12.144$ kip-ft. for Class "Especial Heavy."

Maximum Moment at "e."

Wheel 13 at *e*
$G/n = 431.1 \ / \ 7 = 61.6$
$G_l/m = 229.5 \ / \ 4 = 57.38$ or $229.5 \ / \ 4 = 63.23$
Wheel 13, therefore, gives a maximum, but wheels 12 and 14 do not.
$R_l = 34{,}988 \ / \ 175 = 200$ kips
$M = 200 \times 100 - 9148 = 10{,}852$ kip-ft. for Class "I"
$= 10{,}852 \times 1.111 = 12{,}058$ kip-ft. for Class "Especial Heavy."

Maximum Shear in Panel "ab."

Criterion for position of loading giving maximum shear in panel *ab* gives same position as for maximum moment at *b*. Wheel 4 is, therefore, placed at *b* as shown in Fig. 18.
$R_l = 249$ kips
$V = 249 - 569.7 \ / \ 25 = 226.2$ kips for Class "I"
$= 226.2 \times 1.111 = 251.3$ kips for Class "Especial Heavy."

Maximum Shear in Panel "bc."

Wheel 4 at *c*
$G/n = 417.6 \ / \ 7 = 59.66$
$G_2 = 58.95$ or 82.35.
Condition for a maximum shear. Wheel 3 at *c* also gives the condition for a maximum, but the resulting shear is slightly less than for wheel 4 at *c*. For the latter,
$R_l = 32{,}441 \ / \ 175 = 185.5$ kips.
$V = 185.5 - 569.7 \ / \ 25 = 162.7$ kips for Class "I"
$= 162.7 \times 1.111 = 180.8$ kips for Class "Especial Heavy."

Maximum Shear in Panel "cd."

Wheel 3 at *d*
$G/n = 350.1 \ / \ 7 = 50.1$
$G_2 = 35.55$ or 58.95
Condition for a maximum shear. Wheels 2 and 4 do not give a maximum.
$R_l = 29{,}260 \ / \ 175 = 119.5$ kips
$V = 119.5 - 274.95 \ / \ 25 = 108.5$ kips for Class "I."
$= 108.5 \times 1.111 = 120.6$ kips for Class "Especial Heavy."

Maximum Shear in Panel "de."

Wheel 3 at e
$G/n = 276.3 / 7 = 39.47$ or $292.50 / 7 = 41.79$
$G_2 = 35.55$ or 58.95.
Condition for a maximum. Wheels 2 and 4 do not give a maximum.
$R_1 = 12{,}899 / 175 = 73.7$ kips
$V\ 73.7 - 274.95 / 25 = 62.7$ kips for Class "I."
$= 62.7 \times .1111 = 69.7$ kips for Class "Especial Heavy."

Maximum Shear in panel "ef."

Wheel 2 at f
$G/n = 182.7 / 7 = 26.1$
$G_2 = 12.15$ or 35.55
Condition for a maximum. Wheel 3 does not give a maximum.
$R_1 = 5{,}873 / 175 = 33.6$ kips
$V = 33.6 - 97.2 / 25 = 29.7$ kips for Class "I."
$= 29.7 \times 1.111 = 33.0$ kips for Class "Especial Heavy."

Maximum Shear in panel "fg."

Wheel 2 at g
$V = 8.7$ kips for Class "Especial Heavy."

TABLE OF MAXIMUM LIVE LOAD SHEARS AND MOMENTS IN ONE TRUSS

Shears.

PANEL	WHEEL AT PANEL POINT TO RIGHT OF PANEL	MAXIMUM SHEAR IN PANEL, IN LB.
ab	4	251,300
bc	4	180,800
cd	3	120,600
de	3	69,700
ef	2	33,000
fg	2	8,700

Moments.

PANEL POINT	WHEEL AT PANEL POINT	MOMENT IN FT.-LB.
b	4	6,283,000
c	7	10,119,000
d	11	12,144,000
e	13	12,058,000

Members Requiring Special Consideration

Chord Segments "DE" and "de."

Maximum live-load stresses in DE and de are not the same, if *either* diagonal in panel de is under stress. Stress in de is then always less than in DE. Exact stresses will depend on whether De or dE is under stress when loading is in position for the heaviest moments at the panel points on either side of the centre line.

For wheel 11 at d, Fig. 19 (a), shear in panel de = — 11.6 kips.
For wheel 13 at e, Fig. 19 (b), shear in panel de = + 7.4 kips.

For wheel 11 at d, M at d = 12,144,000 ft.-lb.
and stress in DE = 12,144,000 / 31 = 392,000 lb.
For wheel 11 at d, M at E = 11,843,000 ft.-lb.
and stress in de = 11,843,000 / 31 = 382,000 lb.
For wheel 13 at e, M at D = 11,873,000 ft.-lb.
and stress in de = 11,873,000 / 31 = 383,000 lb.
For wheel 13 at e, M at e = 12,058,000 ft.-lb.
and stress in DE = 12,058,000 / 31 = 389,000 lb.

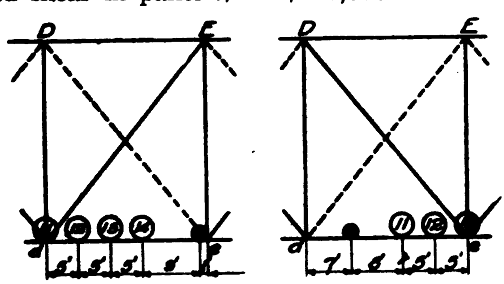
The maximum stress in DE is, therefore, 392,000 lb., and in de it is 383,000 lb.

Hip-hanger "Bb."

Live-load stress in member equals floor-beam concentration from one rail = 99,200 lb., without consideration of impact.

Counters "cD" and "Ef."

Dead load shear in panel ef = — 42,600 lb.
Live load shear in panel ef = + 33,000 lb.



(a) (b)

FIG. 19—POSITIONS OF LIVE LOAD FOR MAXIMUM CHORD STRESSES IN PANEL de

Since an impact allowance is added to the live load stress, the sum will exceed the dead load stress and counters are necessary.

Comparing dead and live load shears in panel *fg*, counters are found unnecessary.

Collision Strut "a″b."

The stress in this arises from an assumed horizontal thrust of 50,000 lb. acting in the plane of the truss and 4 ft. 6 in. above the base of the rail. Stress = 57,000 lb.

TABLE OF LIVE LOAD STRESSES IN TRUSS

Stress in member *aB*	=	251,300 × 1.284	=	322,000 lb.
Stress in member *Bc*	=	180,800 × 1.284	=	232,000 lb.
Stress in member *Cd*	=	120,600 × 1.284	=	155,000 lb.
Stress in member *De*	=	69,700 × 1.284	=	89,500 lb.
Stress in member *Bb*	=			99,200 lb.
Stress in member *Cc*	=	120,600 × 1	=	120,600 lb.
Stress in member *Ed*	=	69,700 × 1	=	69,700 lb.
Stress in member *cD*	=	33,000 × 1.284	=	42,400 lb.

Stress in member *abc*	=	6,283,000 / 31	=	203,000 lb.
Stress in member *BC or cd*	=	10,119,000 / 31	=	326,000 lb.
Stress in member *CDEF*	=	12,144,000 / 31	=	392,000 lb.
Stress in member *de*	=	11,873,000 / 31	=	383,000 lb.

WIND STRESSES IN TRUSSES AND IN LATERAL SYSTEMS

Assumed Loads.

Wind pressure in plane of top lateral system = 200 lb. per lineal ft., moving load.

Wind pressure of 600 lb. per lineal ft., moving load, applied in a horizontal plane at a height of 8 ft. above base of rail.

Pressure of wind on bridge (and train) gives rise to four different effects:

(1) Effect of horizontal pressure on top lateral system and top chords of main trusses.

(2) Effect of horizontal pressure 8 ft. above base of rail on bottom lateral system and bottom chords of main trusses.

(3) Overturning effect on trusses of wind pressure in plane of top lateral system.

(4) Overturning effect on trusses of wind pressure on train, considered as applied 8 ft. above base of rail.

(1) Wind Pressure in Plane of Top Laterals.

Lateral truss is 18 ft. deep and consists of five panels of 25 ft. each. See Fig. 10.

Panel wind load = 200 × 25 = 5,000 lb.

Length of diagonals = $\{(25)^2 + (18)^2\}^{1/2}$ = 30.8 ft.

Sec. θ' = 30.8 / 18 = 1.71.

Table of Maximum Stresses in Top Lateral System

Shear in panel BC	$= 10{,}000 - 0$		$= 10{,}000$ lb.
Shear in panel CD	$= 5{,}000 \times 6/5$		$= 6{,}000$ lb.
Shear in panel DE	$= 5{,}000 \times 3/5$		$= 3{,}000$ lb.
Moment at C or C'	$= 10{,}000 \times 25$		$= 250{,}000$ ft.-lb.
Moment at D or D'	$= 10{,}000 \times 50 - 5{,}000 \times 25$		$= 375{,}000$ ft.-lb.
Stress in diagonal BC'	$= 10{,}000 \times 1.71$		$= 17{,}100$ lb.
Stress in diagonal CD'	$= 6{,}000 \times 1.71$		$= 10{,}300$ lb.
Stress in diagonal DE'	$= 3{,}000 \times 1.71$		$= 5{,}200$ lb.
Stress in member BC or $C'D'$	$= 250{,}000 / 18$		$= 13{,}900$ lb.
Stress in member CDE or $D'E'$	$= 375{,}000 / 18$		$= 20{,}900$ lb.

(2) *Wind Pressure in Plane of Bottom Laterals.*

Panel wind load $= 600 \times 25 = 15{,}000$ lb.
Sec. $\theta = 1.71$.
See stress sheet, Fig. 10.

Table of Maximum Stresses in Bottom Lateral System

Shear in panel ab	$= 45{,}000 - 0$	$= 45{,}000$ lb.
Shear in panel bc	$= 15{,}000 \times 15/7$	$= 32{,}100$ lb.
Shear in panel cd	$= 15{,}000 \times 10/7$	$= 21{,}400$ lb.
Shear in panel de	$= 15{,}000 \times 6/7$	$= 12{,}900$ lb.
Moment at b or b'	$= 45{,}000 \times 25$	$= 1{,}125{,}000$ ft.-lb.
Moment at c or c'	$= 45{,}000 \times 50 - 15{,}000 \times 25$	$= 1{,}875{,}000$ ft.-lb.
Moment at d or d'	$= 45{,}000 \times 75 - 15{,}000 \times (25 + 50)$	$= 2{,}250{,}000$ ft.-lb.
Stress in diagonal ab'	$= 45{,}000 \times 1.71$	$= 77{,}000$ lb.
Stress in diagonal bc'	$= 32{,}100 \times 1.71$	$= 55{,}000$ lb.
Stress in diagonal cd'	$= 21{,}400 \times 1.71$	$= 36{,}600$ lb.
Stress in diagonal de'	$= 12{,}900 \times 1.71$	$= 22{,}100$ lb.
Stress in member ab or $b'c'$	$= 1{,}125{,}000 / 18$	$= 62{,}500$ lb.
Stress in member bc or $c'd'$	$= 1{,}875{,}000 / 18$	$= 104{,}200$ lb.
Stress in member cde or $d'e'$	$= 2{,}250{,}000 / 18$	$= 125{,}000$ lb.

(3) *Overturning Effect of Wind on Trusses.*

Wind reaction applied at hip-joint by top lateral system = 5,000 × 3 = 15,000 lb.

Vertical reactions at bases of end posts = 15,000 × 31 / 18 = 25,900.

Upward vertical force applied at hip or windward truss by portal and downward vertical force applied at hip of leeward truss = 25,900 lb.

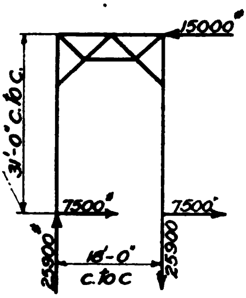


FIG. 20—OVERTURNING EFFECT OF WIND ON TRUSSES

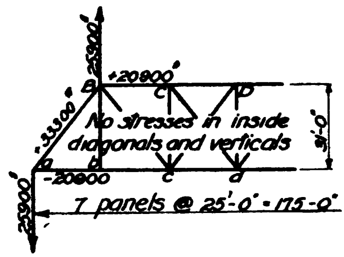


FIG. 21—STRESSES IN WINDWARD TRUSS, DUE TO OVERTURNING EFFECT OF WIND ON TRUSSES

Stresses in windward truss are as shown in Fig. 21. Stresses in leeward truss are of same magnitude but of opposite sign.

(4) *Overturning Effect of Wind on Train.*

Wind force applied 8 ft. above base of rail = 600 lb. per lineal ft.

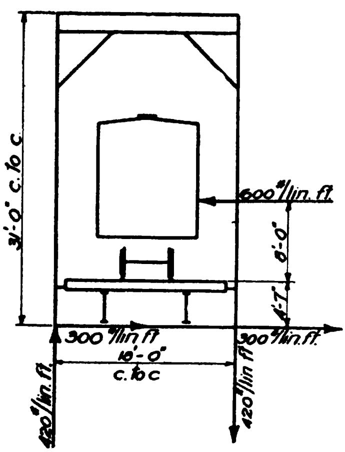


FIG. 22—OVERTURNING EFFECT OF WIND ON TRAIN

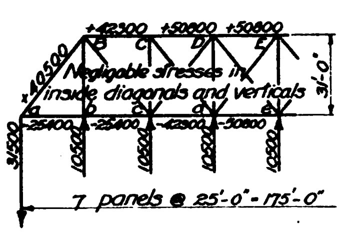


FIG. 23—STRESSES IN WINDWARD TRUSS, DUE TO OVERTURNING EFFECT OF WIND ON TRAIN

Base of rail approximately 4 ft. 7 in. above centre line of bottom chord.

Downward load applied to leeward truss and upward load applied to windward truss due to wind on train.

= 600 × 12.59 / 18 = 420 lb. per lineal ft.

Panel wind load = 420 × 25 = 10,500 lb.

Stresses in windward truss as shown in Fig. 23. Stresses in leeward truss are of same magnitude but of opposite sign. Stresses in all web members excepting end posts are negligible.

WIND STRESSES IN TRUSSES

Windward Truss	Wind on Top Chord	Wind on Bottom Chord	Overturning Effect of wind on trusses	Overturning Effect of wind on train	Total Wind Stress
BC	—13,900		+ 20,900	+ 42,300	+ 49,300
CD	—20,900		+ 20,900	+ 50,800	+ 50,800
DE	—20,900		+ 20,900	+ 50,800	+ 50,800
ab		— 62,500	—20,900	—25,400	—108,800
bc		—104,200	—20,900	—25,400	—150,500
cd		—125,000	—20,900	—42,300	—188,200
de		—125,000	—20,900	—50,800	—196,700
aB			+ 33,300	+ 40,500	+ 73,800
Leeward Truss					
B'C'	0		—20,900	—42,300	— 63,200
C'D'	+ 13,900		—20,900	—50,800	— 57,800
D'E'	+ 20,900		—20,900	—50,800	— 50,800
a'b'		0	+ 20,900	+ 25,400	+ 46,300
b'c'		+ 62,500	+ 20,900	+ 25,400	+ 108,800
c'd'		+ 104,200	+ 20,900	+ 42,300	+ 167,400
d'e'		+ 125,000	+ 20,900	+ 50,800	+ 196,700
a'B'			—33,300	—40,500	— 73,800

STRESSES IN PORTAL

Wind force applied at hip-joint, or at end of top strut of portal = 3 panel loads = 5,000 × 3 = 15,000 lb.

Lower end of each end-post assumed to be fixed, and plane of contra-flexure assumed to be half-way between foot of end-post and knee-brace connection.

Ends of post then assumed to lie in plane of contra-flexure with a horizontal reaction at each = 15,000 / 2 = 7,500 lb.

Bending moment in end-posts at knee-brace connections k and k' = 7,500 × 13.9 = 104,200 ft.-lb.

Force induced in portal strut at top of each post due to this moment = 104,200 / 12 = 8,700 lb.

Stress in $B'S$ = + 8,700 lb.
Stress in BS = — (8,700 + 15,000) = — 23,700 lb.

Horizontal component of force applied to each knee-brace at its lower end, = 8,700 + 7,500 = 16,200 lb.

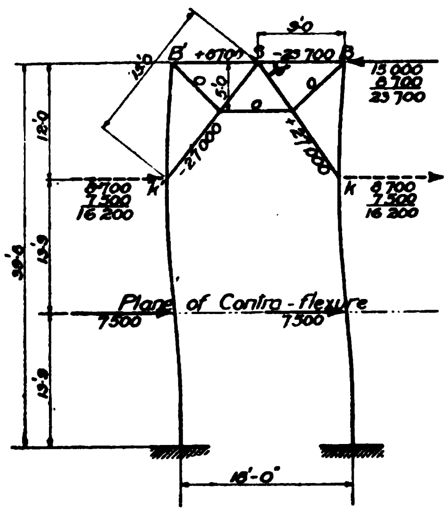


Fig. 24—Stresses in Portal

Stress in knee-brace = 16,200 × sec θ'' = 16,200 × 1,667 = 27,000 lb., tension in windward knee-brace and compression in leeward one.

No calculable stress in stiffening members connecting to centres of knee-braces.

Load on Truss Bearings

Reaction for One Truss.

Maximum live load reaction occurs when wheel 2 is directly over support. Taking moment from moment table and subtracting moment of wheel 1, which is off the span, we get.

L. L. reaction = (50945.4 — 12.15 × 183) / 175 = 279 kips
for Class "I"
= 279 × 1.111 = 310 kips
for Class "Especial Heavy"

D. L. = 42,600 × 3½	= 149,100 lb.
L. L. =	= 310,000 lb.
Imp. = $(310{,}000)^2 / (310{,}000 + 149{,}100)$	= 209,000 lb.
	668,100 lb.

Proportioning of Truss Members

End Post "aB."

Designed for direct stresses due to dead load, live load, impact,

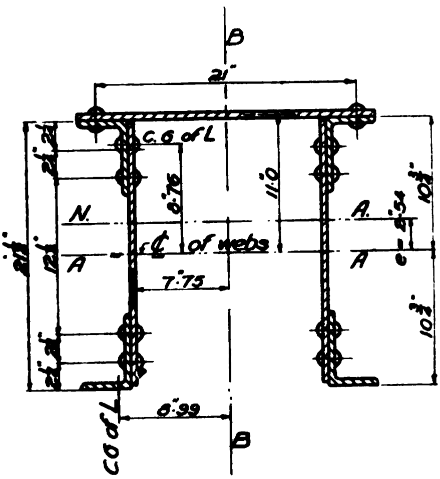


Fig. 25—Section of End Post, *aB*

and wind plus a wind moment in plane of portal of 104,200 ft.-lb. or 1,250,400 in.-lb.

Unsupported width of web plates, between rivet lines to be not over 30 times the thickness of the plate and of cover plate not over 40 times the thickness of the plate.

Assume as section

1 cover plate, 24 × ½ in.	= 12.00 sq. in.
2 web plates, 21 × ½ in.	= 21.00 sq. in.
4 angles, 6 × 4 × ½ in.	= 19.00 sq. in.
Total	= 52.00 sq. in.

arranged as shown in Fig. 25.

For position of centre of gravity, take moments about centre line of webs, or axis AA.

Part	*Area* $= A$	*Arm*	*Statical Moment*
2 web plates	21.00	0	0
2 top angles	9.50	$+ 8.76$	0
2 bottom angles	9.50	$- 8.76$	
Cover plate	12.00	$+ 11.00$	$= 132.0$
	52.00		132.0

Eccentricity $e = 132 / 52 = 2.54$ in.

Moment of inertia about the neutral axis, $N.A. = I_{NA} = I_{NA} - Ae^2$ is approximately as follows:

$I_{AA} =$

Two web plates $= 2 \times 385.88$	$= 772$
Four angles $= 4\,[17.40 + 4.75 \times (8.76)^2]$	$= 1526$
One cover plate $= 0 + 12.0 \times (11.0)^2$	$= 1452$
	3750
$Ae^2 = 52.0 \times (2.54)^2$	$= 335$
$I_{NA} =$	3415

$r_{NA} = (3415 / 52)^{1/2} = 8.1$ in.

$I_{BB} =$

Two web plates $= 2\,[0 + 10.5 \times (7.75)^2]$	$= 1261$
Four angles $= 4\,[6.27 + 4.75 \times (8.99)^2]$	$= 1561$
One cover plate	$= 576$
	3398

$r_{BB} = (3398 / 52)^{1/2} = 8.1$ in.

Direct stresses in end post as follows:

D. L	$= -$ 164,200 lb.
L. L.	$= -$ 322,000 lb.
Imp. $= (322{,}000)^2 / (322{,}000 + 164{,}200)$	$= -$ 213,000 lb.
	$-$ 699,200 lb.
Wind	$-$ 73,800 lb.
	$-$ 773,000 lb.

There are four cases to be considered

(1) Strength in plane of truss, wind not acting,
(2) Strength in plane of portal, wind not acting,
(3) Strength in plane of truss, wind acting,
(4) Strength in plane of portal, wind acting.

(*1*) *Plane of truss no wind.*

Unsupported length = distance from collision strut connection to hip joint = 39.8 — 11.5 = 28.3 ft. = 340 in.

$$l/r = 340 / 8.1 = 41.9$$

Allowable compressive stress, from formula I. of the specifications, $p = 16,000 \div (1 + l^2 / 18,000r^2) = 14,600$ lb. per sq. in.

Area required = 699,200 / 14,600 = 47.9 sq. in. Section is ample for this.

(*2*) *Plane of portal; no wind.*

Unsupported length = distance from foot of post to knee brace connection = 39.8 — 12.0 = 27.8 ft. = 334 in.

$$l/r = 334 / 8.1 = 41.3$$

Since this is less than l/r for case (1), the post is stronger in this plane than in the plane of the truss.

(*3*) *Plane of truss: wind acting.*

Since the direct wind stress is less than 25% of the sum of the dead load, live load and impact stresses, this case need not be considered further.

(*4*) *Plane of portal; wind acting.*

The post is most seriously stressed at the knee-brace connection, but on account of support of post in the plane of the portal by the knee-brace, the allowable compressive stress need not be reduced by a column formula. This stress, when wind is considered may be 20,000 lb. per sq. in.

Area required for direct stress
$= 773,000 / 20,000 \qquad = 38.65$ sq. in.

Area required for bending
$My_1 / r^2p = (1,250,400 \times 12) / \{(8.1)^2 \times 20,000\} \qquad = 11.43$ sq. in.

50.08 sq. in.

The assumed section is, therefore, sufficient.

Top Chord "BC."

The total stress is as follows:

D. L.		— 171,800 lb.
L. L.		— 326,000 lb.
Imp. = $(326{,}000)^2 / (326{,}000 + 171{,}800)$	=	— 213,000
		— 710,800 lb.

Maximum wind stress is 63,200 lb., or less than 25% of the sum of the dead load, live load and impact stresses and, therefore, need not be considered.

Assume as section

1 cover plate, 24 × ½ in.	=	12.00 sq. in.
2 web plates, 21 × ½ in.	=	21.00 sq. in.
4 angles, 6 × 4 × 7/16 in.	=	16.76 sq. in.
Total		49.76 sq. in.

The radii of gyration may, with sufficient accuracy, be assumed the same as for the end-post aB. Then

$$l/r = 300 / 8.1 = 37$$

From the column formula for fixed ends, $p = 16{,}000 \div (1 + l^2 / 18{,}000\, r^2) = 14{,}850$ lb. per sq. in.

Area required = 710,800 / 14,850 = 47.9 sq. in. The section is therefore ample.

Top Chord "CDEF."

Total stress, from stress sheet = 855,000 lb. Wind stress below the 25 per cent. limit. Assume as section

1 cover plate, 24 × ½ in.	=	12.00 sq. in.
2 web plates, 21 × ⅝ in.	=	26.25 sq. in.
4 angles, 6 × 4 × ½ in.	=	19.00 sq. in.
Total		57.25 sq. in.

Radii of gyration will be slightly less than for end-post. Assume r in each direction to be 8.0 in.

$$l/r = 300 / 8 = 37.5$$

From the formula for fixed ends, p = 14,800 lb. per sq. in.

Area required = 855,000 / 14,800 = 57.8 sq. in.

The assumed section is sufficiently near the requirement.

Intermediate Post "Cc."

Total stress = — 259,200 lb.

Width at right angles to plane of truss must be such as to

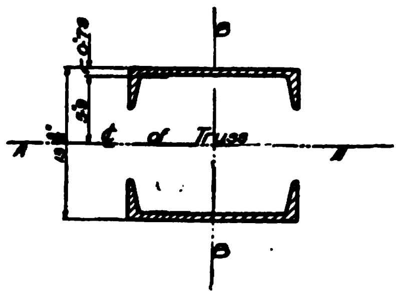


FIG. 26—SECTION OF INTERMEDIATE POST *Cc*

fit inside web plates of the top chord, allowing for two 5/8-in. gusset plates and clearance. Assume a width of 13 3/8 in. out to out.

Assuming intermediate posts to be free to turn at both ends, and a working stress of about 10,000 lb. per sq. in., area required = 259,200 / 10,000 = 25.9 sq. in. approximately. Try two 15-in. channels at 45 lb., latticed on flanges, giving an area of 26.48 sq. in. and spaced 13 3/8 in. back to back, as shown in Fig. 26.

$I_{AA} = 2\,[10.29 + 13.24 \times (5.9)^2] = 943$ approx.

$r_{AA} = (943 / 26.48)^{1/2} = 5.96$ in.

From tables, $r_{BB} = 5.32$ in.

$$l/r = 31 \times 12 / 5.32 = 70$$

$p = 10{,}360$ lb. per sq. in., from the formula for columns with ends free to turn.

Area required = 259,200 / 10,360 = 25.0 sq. in. Assumed section is therefore adequate.

Intermediate Post "Dd".

Total stress = —141,700 lb.

Assume two 15-in channels at 33 lb. = 19.80 sq. in., spaced 13 3/8 in. back to back and latticed on flanges.

Least $r = 5.62$ in.

$l/r = 31 \times 12 / 5.62 = 66$

$p = 10{,}780$ lb. per sq. in., assuming ends free to turn.

Area required = 141,700 / 10,780 = 13.16 sq. in. Assumed section is adequate.

Hip Hanger "Bb."

Total stress as follows:

D. L.	+ 28,400 lb.
L. L. = 99,200 × (1.40 — 43 / 200)	+ 118,000 lb.
Imp. = $(118{,}000)^2$ / (118,000 + 28,400) =	+ 95,000 lb.
	+ 241,400 lb.

Area required = 241,400 / 16,000 = 15.1 sq. in.

Use two 15-in. channels at 33 lb. = 19.80 sq. in., gross, or less four 1-in holes out of webs and four out of flanges = 15.7 sq. in. net.

Bottom Chord "ab."

D. L. =	+ 103,000 lb.
L. L. =	+ 203,000 lb.
Imp. = $(203{,}000)^2$ / (203,000 + 103,000)	+ 135,000 lb.
	+ 441,000 lb.

Wind = + 46,300 lb. or less than 25% of the sum of the dead load, live load and impact stresses.

Area required = 441,000 / 16,000 = 27.55 sq. in.

Use 2 plates, 22 × 7/16 in.; less eight 1-in holes	= 15.75 sq. in.
4 angles, 6 × 4 × 3/8 in.; less four 1-in. holes	= 12.94 sq. in.
	28.69 sq. in.

Bottom Chord "bc."

Wind stress less than 25% of sum of dead load, live load and impact stresses. Use same section as for *ab*.

Bottom Chord "cd."

Total stress = + 710,800 lb., not including wind stress, which is below the 25% limit. Area required = 710,800 / 16,000 = 44.4 sq. in. Use

2 plates, 22 × 3/4 in.; less eight 1-in. holes	= 27.0 sq. in.
4 angles, 6 × 4 × 1/2 in.; less four 1-in holes	= 17.0 sq. in.
	44.0 sq. in.

Bottom Chord "de."

Total stress = + 838,000 lb., neglecting wind. Area required = 838,000 / 16,000 = 52.4 sq. in.

Use 2 plates, 22 × ¾ in.; less eight 1-in. holes	= 27.00 sq. in.
2 plates, 10 × ½ in.; less four 1-in. holes	= 8.00 sq. in.
4 angles, 6 × 4 × ½ in.; less four 1-in. holes	= 17.00 sq. in.
	52.00 sq. in.

Diagonal "Bc."

Total stress = + 499,100 lb. neglecting wind. Area required = 499,100 / 16,000 = 31.2 sq. in. Use

4 angles, 6 × 4 × 13/16 in.; less eight 1-in holes	= 23.38 sq. in.
1 plate, 13 × ¾ in.; less two 1-in. holes	= 8.25 sq. in.
	31.63 sq. in.

Diagonal "Cd."

Total stress = + 324,700 lb.

Area required = 324,700 / 16,000 = 20.3 sq. in. Use

4 angles, 6 × 4 × ½ in.; less eight 1-in. holes	= 15.00 sq. in.
1 plate, 13 × ½ in.; less two 1-in. holes	= 5.50 sq. in.
	20.50 sq. in.

Diagonal "De."

Total stress = + 179,000 lb.

Area required = 179,000 / 16,000 = 11.20 sq. in.

Use 4 angles, 6 × 4 × ⅜ in., battened.

Less eight 1-in. holes = 11.44 sq. in.

Counter "cD."

Consider main diagonal *Cd* not acting.

Dead load stress in *cD* then = 42,600 × 1.284 = 54,700 lb. Specification requires that only 70% of this be considered as counteracting live load stress. Impact = net live load stress.

D. L. = — 54,700 × 7/10	= — 38,300 lb.
L. L.	= + 42,400 lb.
Imp.	= + 4,100 lb.
	+ 8,200 lb.

Area required = 8,200 / 16,000 = 0.51 sq. in. Use two angles, $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in.; less two 1-in holes = 4.23 sq. in. net.

Collision Strut "q''b."

Total stress = — 57,000 lb.

Assume two angles, $7 \times 3\frac{1}{2} \times 7/16$ in. = 8.80 sq. in. with short legs turned in and spaced about $13\frac{3}{8}$ in. apart back to back.

$$l/r = 240 / 2.26 = 106.$$

p, for ends free to turn = 7,120.

Area required = 57,000 / 7,120 = 8.01 sq. in. Section sufficient.

Proportioning of Members in Portal

For stresses, see Fig. 24.

Top Strut "BB'".

Compressive stress = 23,700 lb.

Assume two angles, $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. = 4.98 sq. in., arranged as shown on stress sheet. About an axis perpendicular to plane of portal.

$$l/r = 108 / 1.07 = 101$$

About the other axis, l/r is much less than this.

p, from formula for ends free to turn = 7,500 lb. per sq. in.

Area required = 23,700 / 7,500 = 3.16 sq. in. Section sufficient.

Knee Braces.

Maximum stress = 27,000 lb.

Assume two angles, $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. = 4.98 sq. in. arranged in a manner similar to those in the top strut.

About axis perpendicular to plane of portal,

$$l/r = 90 / 1.07 = 84.$$

p = 8970 lb. per sq. in.

Area required = 3.01 sq. in. Section sufficient.

Stiffening Members.

These carry no calculable stress. Use two angles. $3\frac{1}{2} \times 3\frac{1}{2} \times 5/16$ in., latticed.

Proportioning of Intermediate Vertical Bracing

This is not generally figured, but is put in largely by judgment. For both top strut and knee braces l/r should not exceed 120.

Use for top strut, four angles $4 \times 3 \times \frac{3}{8}$ in., laced as shown on stress sheet. About vertical axis.

$$l/r = 216 / 1.90 = 114.$$

For knee braces, use 2 angles, $3\frac{1}{2} \times 3 \times \frac{3}{8}$ in. arranged as indicated on the stress sheet.

$$l/r = 114 / 1.10 = 104.$$

Proportioning of Laterals

Stresses and sections provided are given on stress sheet. Stringer laterals are set by judgment.

Proportioning of Pier Members

Maximum reaction at truss bearings = 668,100 lb.

Required area of bed plate, bearing on concrete = 668,100 / 400 = 1,670 sq. in.

Allowable pressure on rollers per lineal inch = $1200\ D^{\frac{1}{2}}$. Assuming 5-in. rollers, allowable pressure = $1,200\ (5)^{\frac{1}{2}} = 2,680$ lb. per lineal in. Number of lineal inches of rollers required for bearing on shoe and bed plates = 668,100 / 2,680 = 250 in. Use 7 lines of 5-in. rollers, 36 in. shoulder to shoulder = 252 lineal in. To accommodate rollers, allowing $\frac{1}{4}$-in. clearance between rollers and about 5 in. from the centre of the outside roller to edge of plate and 5 in. from shoulder to edge of plate, the shoe and bed plates must be 42×46 in., the latter dimension being parallel to the rollers.

Proportioning of Intermediate Vertical Bracing

This is not generally figured, but is put in largely by judgment. For both top strut and knee braces l/r should not exceed 120.

Use for top strut four angles $4 \times 3 \times \frac{5}{16}$ in., laced as shown on stress sheet. About vertical axis,

$$l/r = 216 / 1.90 = 114.$$

For knee braces, use 2 angles $3\frac{1}{2} \times 3 \times \frac{3}{8}$ in. arranged as indicated on the stress sheet.

$$l/r = 114 / 1.10 = 104.$$

Proportioning of Laterals

Stresses and sections provided are given on stress sheet. Sections of laterals are set by judgment.

Proportioning of Pier Members

Maximum reaction at truss bearings = 668,100 lb.

Required area of bed plate, bearing on concrete = 668,100 / 400 = 1,670 sq. in.

Allowable pressure on rollers per lineal inch = $1200\ D^{\frac{1}{2}}$. Assuming 5-in. rollers, allowable pressure = $1,200\ (5)^{\frac{1}{2}}$ = 2,680 lb. per lineal in. Number of lineal inches of rollers required for bearing on shoe and bed plates = 668,100 / 2,680 = 250 in. Use 7 lines of 5-in. rollers, 36 in. shoulder to shoulder = 252 lineal in. To accommodate rollers, allowing $\frac{1}{2}$-in. clearance between rollers and about 5 in. from the centre of the outside roller to edge of plate and 5 in. from shoulder to edge of plate, the shoe and bed plates must be 42 × 40 in., the latter dimension being parallel to the rollers.